DESIGN FOR POST-FIRE USE: A CASE STUDY IN FIRE RESILIENCE DESIGN

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ABSTRACT

Contemporary society increasingly emphasizes the importance of robust and resilient systems that can maintain their functionality or easily recover after a damaging event. For structural fire safety engineering, this emphasis on post-event performance requires new ways of thinking (and designing). Currently most structural fire safety requirements are intended to provide sufficient time for building occupants and fire and rescue services to evacuate, with a clear focus on life safety. As part of the trend towards resilience-based design, structural fire safety engineering needs to begin to explicitly consider the post-fire damage, usability, and reparability of structures. To highlight the complexity of this area, and to illustrate some of the key concepts that ought to be considered, this paper presents some of the issues around the practicality of a design concept of 'design for post-fire use' using a simple example of a cantilever concrete slab exposed to fire from below. The post-fire load bearing capacity of the slab is evaluated considering exposure to a Eurocode parametric heating curve, and structural fragility curves are derived to indicate the *a priori* probability that the fire affected slab will need structural strengthening to allow for unrestricted continued use after a fully developed fire in the compartment below. The fragility curves show that in this case a minor reduction in cantilever length can significantly reduce the probability of requiring strengthening post-fire, and conceptually demonstrates that incremental changes in design and construction may significantly improve robustness/resilience with respect to fire.

NOMENCLATURE

h	slab thickness [mm]
l	cantilever slab length [m]
M_R	residual hogging capacity per unit width [kNm/m]
P_f	probability of failure [-]
q_F	fire load density [MJ/m^2]
$q_{F,lim}$	maximum fire load density for which $q_{k,max} \geq q_{k,req}$ [MJ/m^2]
$q_{F,nom}$	nominal (mean) fire load density [MJ/m^2]
$q_{k,max}$	maximum allowable characteristic value of the imposed load (post-fire) [kN/m^2]
$q_{k,req}$	required characteristic value of the imposed load (post-fire) [kN/m^2]
V	coefficient of variation [-]
β	reliability index [-]
μ	mean value
σ	standard deviation

1. INTRODUCTION

Structural fire safety requirements incorporated in current guidance documents clearly focus on Life Safety. Consequently, the post-fire usability of structures is not typically ensured, and little *a priori* consideration is generally given to structural recovery after fire. However, societal expectations are beginning to shift towards expecting rapid recovery and limited loss of functionality after a damaging event [1], [2]. For structural fire safety, this focus on *fire resilience* may translate to one or more of the following qualitative targets for structural performance after a fire:

- rapid functional recovery (limited downtime)
- limited loss of functionality (in area or severity, i.e. no disproportionate loss of functionality)
- rapid and cost-effective recovery (reparability)

These qualitative targets relate closely to the 'results of resilience' identified by Bruneau et al. [4]: less damage (i.e. robustness), faster recovery, and lower consequences.

Whether any or more of the performance targets above are relevant for current structural fire design currently depends on considerations from the building developer and on possible legal or regulatory requirements. In a true Performance Based Design however, the targets of the (structural) fire safety strategy should be determined through consultation with all relevant stakeholders [3], to the extent possible.

For many structures, a high degree of resilience to fire is inherent to the structural system and material characteristics. For example, experience has shown that many building fires are not so severe as to threaten the stability of concrete structures during fire, and high potential for post-fire usability may exist.

It may also be expected that comparatively small additional investments in the design and construction phases could result in sizeable benefits with respect to the post-fire performance. This paper examines the above expectation using the specific example of the post-fire load bearing capacity of a concrete cantilever slab. This very simple example case represents but one aspect of the larger issue of resilience-based structural fire design, but it illustrates the potential for explicitly applying quantified resilience and robustness considerations within structural fire safety practice.

The paper begins in Section 2 with a presentation of the proposed concept of ‘design for post-fire use’, the assumptions and hypotheses underlying this concept, and an introduction to the cantilever slab example case used. Section 3 explains how the residual post-fire load bearing capacity is evaluated in this example, and Section 4 applies the obtained results to make an ‘a-priori post-fire assessment’, deriving fragility curves for the post-fire usability of the slab as a function of the nominal fire load density in the compartment below. Section 5 illustrates how these results can be considered in design decisions, and conclusions are given in Section 6.

2. DESIGN FOR POST-FIRE USE

2.1 CONCEPT

Bocchini et al. [2] state that resilience for civil engineering infrastructure is associated with the ability to deliver a certain level of functionality even after occurrence of an extreme event, and also to quickly recover the lost functionality. Robustness considerations are therefore an integral component of a resilience design strategy [4].

While resilience can be achieved, for example, through fast and inexpensive repair methods and supporting organizational measures, a very basic way to obtain (a degree of) resilience is by avoiding damage in the first place (i.e. through robustness [2] or reducing ignition sources). ‘Design for post-fire use’ is proposed as an easy-to-understand (specific) application of resilience with respect to fire.

It is noteworthy that designing for post-fire usability does not necessarily imply that a structure cannot exhibit any reduction in capacity or increased fire-induced deflections, but rather that the qualitative goals listed in the Introduction are (to a large degree) met through the initial design of the structure itself rather than through post-fire repairs/interventions.

2.2 ASSUMPTIONS AND HYPOTHESES

It is assumed herein that the post-fire usability and reparability of the structure depends (at least in part) on the residual load-bearing capacity at the ultimate limit state. This is not meant to imply that serviceability limit states have no influence on the post-fire performance of the structure. Rather, it is assumed that increased deflections or cracking post-fire do not by themselves hinder the continued use of the elements in question.

For example, in a residential building when a fully developed fire occurs in one of the flats, the ceiling of the fire compartment functions as the floor of the apartment above and may exhibit additional deflections and cracks post-fire. However, if the load-bearing capacity of the slab is sufficient for continued use, the flat above the fire compartment can (in principle) be used (possibly with some very minor inconvenience or aesthetic issues) while insurers, engineers, and contractors discuss possible remedial actions.

Furthermore, post-fire residual load-bearing capacity is beneficial for undertaking any repair works, i.e. ensuring access to the structural elements and widening the range of possible cost-effective repair actions.

2.3 EXAMPLE CASE DESCRIPTION

The simple cantilever slab of Figure 1 is used as an example case for the remainder of this paper, but the presented concepts can readily be applied to e.g. simply supported slabs as well. The slab is subjected to a uniformly distributed permanent load g and imposed load q. The cross-sectional characteristics of the slab are given in Table 1. Since the slab is a cantilever only top steel reinforcement is considered, with nominal area per unit width given by Eq. (1), where the slab width b = 1000mm. The length l of the cantilever is the primary design variable considered in the example.

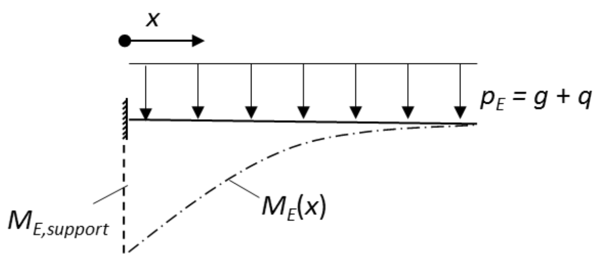

Figure 1: Example case cantilever slab

Table 1: Basic variables for the concrete slab, and stochastic description, based on [12]

Symbol	Property	μ	V
$f_{c,20}$ [MPa]	20°C concrete compressive strength (f_{ck} = 30 MPa)	42.9	0.15
$f_{y,20°C}$ [MPa]	20°C reinforcement yield stress (f_{yk} = 500 MPa)	581.4	0.07
a [mm]	rebar axis distance to top surface slab	40	0.125
h [mm]	slab thickness	200	0.025
A_s [mm^2]	top reinforcement area	$1.02A_{s,nom}$	0.02
s [mm]	rebar axis spacing	100	-
Ø [mm]	rebar diameter	10	-

$$A_{s,nom} = \frac{\pi \varnothing^2 b}{4s} \tag{1}$$

Only the assumed mean value μ and coefficient of variation V have been given in Table 1, since the distributions describing the stochastic variables in Table 1 are not required for evaluating the residual load-bearing capacity when applying the methods discussed herein.
The Eurocode parametric fire approach [5] is used to approximately characterize the potential fire exposure (fire exposure from below). The fire load density q_F is also used as an investigated parameter, all other parameters defining the heating curve are given in Table 2. The opening factor $O = 0.04\text{m}^{1/2}$ has been found to result in the lowest expected residual capacity and the most relative uncertainty [9]. This value will be used further in the absence of a reliable method to estimate glass breakage in fire. As stated in [6], the specific compartment size in Table 2 does not result in a loss of generality, since the same fire curve may apply to other compartments via transformation of the fire load density.
For completeness, Table 3 gives the stochastic distributions for other load and reliability parameters considered.

Table 2: Parameters defining the Eurocode parametric fire curve

Symbol	Property	*Value*
A_f [m^2]	Fire compartment floor area (square compartment)	400
H [m]	Fire compartment height	3
O [m$^{1/2}$]	Opening factor	0.04
b [J/m^{2}s$^{1/2}$K]	Enclosure thermal responsiveness	1700
t_{lim} [min]	Time of maximum compartment temperature when fuel controlled	20*

*medium fire growth

Table 3: Probabilistic models load and resistance variables, based on [12]

Symbol	Property	*Distr.*	μ	V
K_R [-]	model uncertainty for the resistance effect	LN	1.1	0.10
K_E [-]	model uncertainty for the load effect	LN	1.0	0.10
Q [*]	imposed load effect	Gumbel	$0.6Q_k$	0.35
G [*]	slab thickness	N	G_k	0.10
q_F [MJ/m^2]	fire load density	Gumbel	$q_{F,nom}$	0.30

* limit state dependent (e.g. [kNm] for bending)

3. POST-FIRE LOAD CAPACITY

3.1 RESIDUAL HOGGING CAPACITY

Evaluating the post-fire load bearing capacity of the example slab requires knowledge of the residual hogging capacity M_R. Evaluating M_R is non-trivial since fire exposure results in a nonlinear profile of maximum temperatures across the depth of the slab and heating of the concrete material above a given value may result in permanent loss of strength. The reinforcement may also lose a portion of its original capacity when heated above 550-600°C [7], [8].
Numerical evaluations of the residual hogging capacity M_R for the slab in Table 1 are given in [9]. Based on this analysis the analytical formula Eq. (2) has been proposed. This equation is based on the limiting isotherm concept and the simplified force diagram shown in Figure 2, with i_θ being the depth of the limiting isotherm. In accordance with the limiting isotherm concept, the concrete material heated to temperatures above the limiting isotherm is considered not to contribute to the load bearing capacity, while cooler concrete is considered to contribute with its original 20°C strength. In [9] a limiting isotherm of 600°C is shown to be the most appropriate value for evaluating the residual hogging capacity of solid concrete slabs.

$$M_R = A_s f_{y,20}\left(h - i_\theta - a - \frac{A_s f_{y,20}}{2 f_{c,20} b}\right) \tag{2}$$

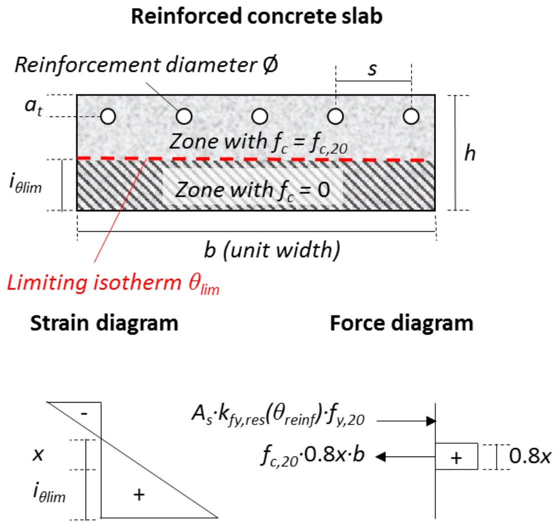

Figure 2: Concept limiting isotherm method and simplified strain and force diagram.

It is noteworthy that no reduction factor for the reinforcement yield stress is included in Eq. (2). This is because limited heating of the top reinforcement will occur under even the most severe credible fire exposures (e.g. corresponding to an ISO 834 standard fire of 360min).

The simplified formula in Eq. (2) results in a deviation with respect to a full numerical cross-sectional model; this deviation has been evaluated in [9] and a lognormal model correction factor K_M has been proposed, which is defined as the ratio between the simplified model of Eq. (2) and a full numerical model, with mean 0.95 and coefficient of variation 0.05 based on Monte Carlo simulations.

3.2 MAX ALLOWABLE IMPOSED LOAD

Eq. (2) gives an assessment of the residual hogging capacity M_R, but this assessment does not clarify which load can be safely carried by the slab. The question of safety is of primary importance, and is at the centre of the current Eurocode design format. Eurocode 0 (EN 1990) [10] specifies that the load effect E can have only a small probability of exceeding the resistance effect R. Defining situations with $Z = R - E < 0$ as 'failure', the Eurocode criterion for a sufficiently safe load carrying situation is given by Eq. (3), where $P[.]$ is the probability operator, P_f is the probability of failure, and β is the corresponding 'reliability index', defined as a transformation of P_f through Φ, the cumulative standard normal distribution function. The left hand side of the inequality in Eq. (3) refers to the failure probability assessed for a specific limit state design, while the right hand side refers to the general safety target specified in EN 1990. Note that the acceptance criterion in Eq. (3) is evaluated for a given reference period t_{ref}. For t_{ref} = 50 years and moderate (i.e. normal) consequences of failure, EN 1990 specifies a target reliability index $\beta_t = 3.8$.

$$P_f = P\left[R - E < 0\right] = \Phi\left(-\beta\right) \le \Phi\left(-\beta_t\right) = P_{f,t} \tag{3}$$

The criterion in Eq. (3) applies directly to the design of new structures through EN 1990 [10]. A similar safety criterion must naturally apply for the post-fire assessment (and continued use) of structural elements to maintain compatibility with the Eurocode design philosophy. By taking into account a reduced remaining lifetime of the existing structure, the target reliability index β_t can (in principle) be smaller than the target for new structures [11]. However in the current work it is reasonably assumed that the target reliability index of 3.8 (50 year reference period) still applies for defining safe continued use of a structure after fire exposure, thus ensuring that no discrepancy exists between the target safety level of newly built structures as compared with structures considered fit for continued use after a fire.

To apply Eq. (3) for evaluating the safe post-fire load acting on the cantilever slab, the general limit state equation $Z = R - E$ is specified to the hogging moment bending limit state. of Eq. (4), with K_R being the model uncertainty for the resistance effect, K_M the lognormal model correction factor introduced above, which corrects for the simplifications introduced by the analytical approximation in Eq. (2), M_R the residual hogging capacity, evaluated by Eq. (2), K_E the model uncertainty for the load effect, M_G the bending moment induced by the permanent load effect, and M_Q the bending moment induced by the imposed load effect. Distributions for the model uncertainties and load effects have already been given in Table 3.

$$Z = K_R \frac{M_R}{K_M} - K_E\left(M_G + M_Q\right) \tag{4}$$

Since the permanent load effect is defined by the self-weight of the slab and its finishes, and since imposed load effects listed in guidance documents are defined by their characteristic values, the question of the post-fire load bearing capacity of the cantilever slab translates to a question regarding the maximum allowable characteristic value $q_{k,max}$ of the imposed load (inducing the bending moment M_Q). This maximum allowable value is defined through the safety criterion in Eq. (3), the limit state in Eq. (4), and the probabilistic models of Table 3; however evaluating these equations requires application of reliability methods, which is challenging from a practical perspective. The simple methodology of the ReAssess method was applied for evaluating Eqs. (3) and (4), and for determining the (reliability-based) maximum allowable characteristic value $q_{k,max}$ of the imposed load after fire exposure.

3.3 REASSESS METHOD

The ReAssess method has been developed in [6] for evaluating the post-fire maximum allowable imposed load, and provides a straightforward method to evaluate Eq. (3). Specifically, the method uses a pre-calculated Assessment Interaction Diagram (AID), visualizing all combinations *G, Q* and *R** fulfilling the criterion of Eq. (3). *G* and *Q* refer to the permanent and imposed load effects, without specific requirements regarding the type of limit state (the AID is, in principle, generally applicable – see [6] for details). The notation *R** denotes the 'combined resistance effect', which combines the 'crude' resistance *R* with all model uncertainties and correcting parameters. Referring to Eq. (4), *R** is given by Eq. (5). Since all model uncertainties follow a lognormal distribution (see Table 3), these can be analytically combined in the total model uncertainty K_T, as in Eq. (5).
Omitting '*' for simplicity, the AID gives the maximum allowable load ratio χ_{max} defined by Eq. (6), for a given input of μ_R / μ_G (the ratio of the expected value of the combined resistance effect to the expected value of the permanent load effect) and V_R (the coefficient of variation for the combined resistance effect). The second equality in Eq. (6) applies specifically to the example case of the cantilever slab. The AID for β_t = 3.8 and a 50 year reference period is given in Figure 3. Note that this AID applies for a lognormal combined resistance effect. Considering the lognormality of K_T, applying the AID requires that M_R is lognormal; this has been verified for the residual hogging capacity in [9].

$$R^* = \frac{K_R}{K_E K_M} M_R = K_T M_R \tag{5}$$

$$\chi_{\max} = \frac{Q_{k,\max}}{G_k + Q_{k,\max}} = \frac{q_{k,\max}}{g_k + q_{k,\max}} \tag{6}$$

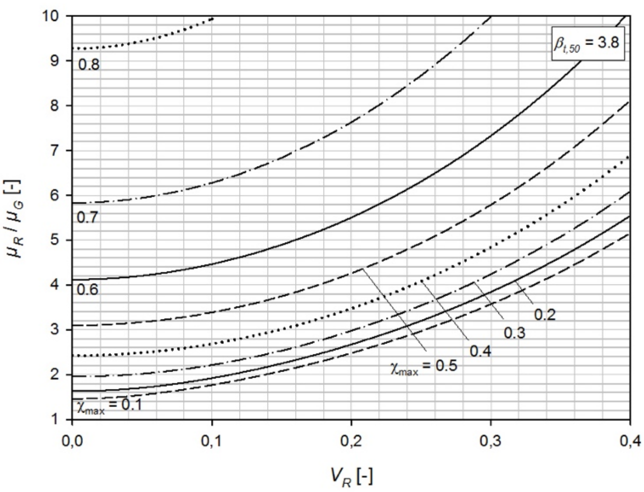

Figure 3: Assessment Interaction Diagram (AID) for β_t = 3.8 and a 50 year reference period.

Based on [12], $\mu_G = G_k$. For the cantilever slab this represents an evaluation of the bending moment introduced by g_k as:

$$\mu_G = G_k = M_{Gk} = \frac{g_k l^2}{2} \tag{7}$$

The mean μ_R and coefficient of variation V_R of Eq. (5) need to be evaluated for application of the AID. This evaluation is done via Eqs. (8)-(10) using Taylor approximations, with the vector $\boldsymbol{\mu}$ of mean values of the parameters in Table 1, and with associated standard deviations σ_{Xi} (defined by $\sigma = \mu \cdot V$). For Eq. (9), K_T is included in the denominator X_i. Eqs. (8)-(10) are analytical and can be evaluated using spreadsheet calculations.

$$\mu_R \approx \mu_{KT} M_R(\boldsymbol{\mu}) \tag{8}$$

$$\sigma_R \approx \sqrt{\sum_i \left(\frac{\partial (K_T M_R)(\boldsymbol{\mu})}{\partial X_i} \right)^2 \sigma_{Xi}^2} \tag{9}$$

$$V_R = \frac{\sigma_R}{\mu_R} \tag{10}$$

Based on the Taylor approximations above, and the evaluation of μ_G by Eq. (7), the post-fire maximum allowable (reliability-based) characteristic value $q_{k,max}$ of a uniformly distributed imposed load on the cantilever slab with length *l,* is defined directly by knowledge of the depth of the 600°C limiting isotherm i_{600} in Eq. (2). All other parameters in Eqs. (2) and (5) are defined by Table 1 and Table 3. This i_{600} depth can be evaluated by heat transfer calculations as a function of the fire load density q_F, considering the Eurocode parametric fire curve and material models [5] and the parameters given in Table 2.

4. *A PRIORI* POST-FIRE ASSESSMENT

Results for the post-fire allowable load $q_{k,max}$ as a function of the fire load density q_F are given in Figure 4 as a function of the cantilever length *l*. A fire load density of 0MJ/m^2 corresponds to the capacity before fire. Figure 4 shows that no reduction in $q_{k,max}$ is observed for a fire load density below 200MJ/m^2. For larger fire load densities the post-fire $q_{k,max}$ is reduced by exposure to fire, but the gradient of the reduction depends strongly on the length *l* of the cantilever (i.e. on the size of the governing load effect).

The assessment in Figure 4 is *a priori* since it considers the *a priori* values for the stochastic parameters given in Table 1. For an existing slab, the uncertainty of e.g. the concrete compressive strength $f_{c,20}$ and the reinforcement axis position *a* could be reduced by direct measurement. Also, after a fire event the depth of the limiting isotherm i_{600} may be assessed directly by observing the temperature changes inside the concrete [7] or by petrography.

5. EFFECT ON DESIGN DECISIONS

For a given cantilever length *l* and a specified post-fire load bearing capacity $q_{k,req}$ (required for post-fire use or reparability), the results in Figure 4 define a deterministic fire load density $q_{F,lim}$, below which $q_{k,max} \geq q_{k,req}$ and above which $q_{k,max} < q_{k,req}$. Assuming e.g. that l = 3m and $q_{k,req}$ = 2kN/m^2 gives $q_{F,lim}$ = 523.5MJ/m^2. It should be noted that $q_{k,req}$ = 2kN/m^2 corresponds to the imposed load capacity required for residential use according to EN 1991-1-1 [13]. If the post-fire $q_{k,max}$ is lower than 2kN/m^2 the use of the slab would therefore be restricted until the slab could be sufficiently strengthened.

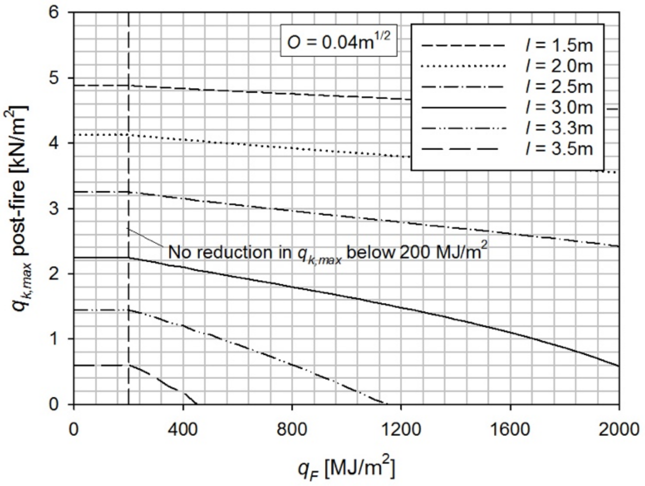

Figure 4: Post-fire $q_{k,max}$ as a function of the fire load density q_F, for different cantilever lengths *l*.

However, for a given occupancy type (in the compartment below the cantilever slab), the fire load density q_F is stochastic and is described by a Gumbel distribution [5], as specified in Table 3. This realization allows evaluation of the probability of requiring structural strengthening for continued use or reparability after a fire as a function of the required $q_{k,req}$. This is done through Eq. (11), with F_{qF} being the cumulative density function for the fire load density and with $q_{F,lim}(q_{k,req})$ determined using Figure 4.

$$P\left[q_{k,\max} < q_{k,req}\right] = P\left[q_F > q_{F,\lim}\left(q_{k,req}\right)\right] = 1 - F_{qF}\left(q_{F,\lim}\left(q_{k,req}\right)\right) \quad (11)$$

For an office compartment with a nominal (mean) fire load density $q_{F,nom}$ of 420MJ/m^2 [5], Eq. (11) suggests a probability 0.178 of requiring structural strengthening (considering $q_{k,req}$ = 2kN/m^2 and l = 3m) after a fully developed parametric fire. For a classroom with $q_{F,nom}$ = 285MJ/m^2 this probability reduces to 0.016. These evaluations clearly relate to *a priori* questions of resilience (or robustness). For example: ‘what is the probability that a fire will result in structural damage requiring (immediate) structural strengthening?’ or ‘what is the probability that a fire will result in a residual capacity too low for reparability?’.

As previously hypothesized, it is possible that minor additional investments or design changes in the construction phase could significantly improve the resilience of the slab with respect to an uncertain fire exposure. This type of *a priori* assessment could therefore prove valuable, especially in situations where the cost of *a posteriori* strengthening is high or where the indirect costs associated with downtime are prohibitive.

Generalizing the above, the fragility curve for $q_{k,req}$ = 2 kN/m^2 is evaluated in Figure 5 as a function of the nominal (mean) fire load density $q_{F,nom}$ for a 3m long cantilever, together with the fragility curves associated with the requirements $q_{k,req}$ = 1.5 kN/m^2 and $q_{k,req}$ = 1.0 kN/m^2. Depending on the application, different consequences may be associated with these or other thresholds. Figure 5 also gives the nominal fire load densities corresponding to a number of occupancy types, as specified in EN 1991-1-2 [5]. Note that the uncertainty on the fire load density q_F is considered via the probabilistic model given in Table 3.

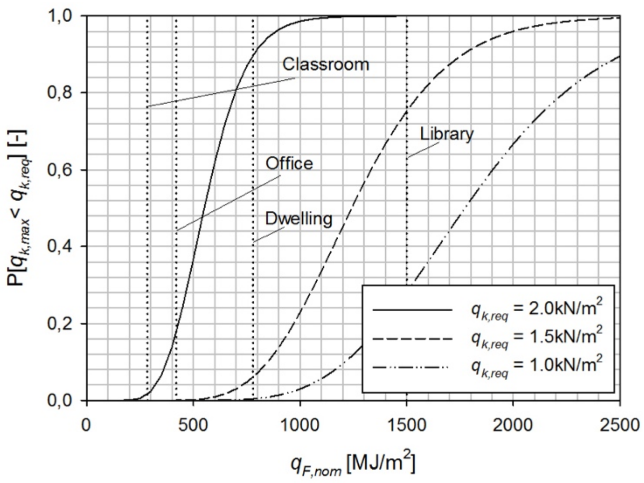

Figure 5: Fragility curves as a function $q_{F,nom}$ for the post-fire condition, for different required loads $q_{k,req}$ and l = 3m.

Based on Figure 5 and considering $q_{k,req}$ = 2kN/m^2 (applicable for normal use of residential dwellings [13]), the 3m long cantilever slab has a low probability of not meeting this load requirement after a fire when the compartment below is used as a classroom. The same configuration however has a much higher probability of requiring strengthening after fire exposure when the compartment is used as a library (i.e. very high probability that $q_{k,max}$ < 2kN/m^2).

Assuming that the cantilever slab is part of a multi-storey apartment building where multiple dwellings are located one above the other, Figure 5 suggests a high probability that the slab will require structural strengthening for continued use after a fire event. This follows from the observation that P[$q_{k,max}$< 2kN/m^2] is about 0.90. Consequently, a fully developed fire in one of the apartments will almost certainly result in usage restrictions for the cantilever slab in the apartment above (i.e. a fire event is likely to necessitate structural strengthening for the safe continued use of the apartment above, despite the fact that this apartment may not have been directly affected by the fire).

As already mentioned, the post-fire performance of the cantilever slab could explicitly be considered in the design phase; i.e. it is possible to design the cantilever slab for post-fire continued use. Considering the case of the apartment building, this design would (within reason) assure the occupants that a fire in another property would not result in a loss of functional usability for their own apartment. Furthermore, for insurance companies this type of assurance may be of interest.

Designing for post-fire use can be achieved, for example, by slightly ‘overdesigning’ the slab for its ‘normal’ loading requirements. Maintaining the slab configuration given in Table 1 and considering $q_{k,req}$ = 2kN/m^2, both prior to and post-fire, this configuration can be considered as an *a priori* overdesign for a cantilever length smaller than 3m (as shown in Figure 4).

Evaluating the fragility curve for $q_{k,req}$ = 2kN/m^2 of Figure 5 as a function of the cantilever length l results in a fragility surface given in Figure 6.

The resilience of the cantilever slab after a fire event is significantly better when applying the slab configuration for a slightly smaller cantilever length. This can be clearly observed in Figure 7, where fragility curves are given which were obtained by intersecting the fragility surface of Figure 6 for the nominal fire load densities q_{nom} corresponding with the occupancy types classroom, office, dwelling and library (from Figure 5). For the residential occupancy, for example, the probability of requiring structural strengthening after a fire event (for reasons of structural safety) can be reduced from 0.90 to less than 0.03 simply by reducing the cantilever length from 3m to 2.8m in this case.

This illustrative result exemplifies the concept of ‘design for post-fire use’; i.e. that a relatively minor design change can yield a significant *a priori* assurance of continued post-fire usability.

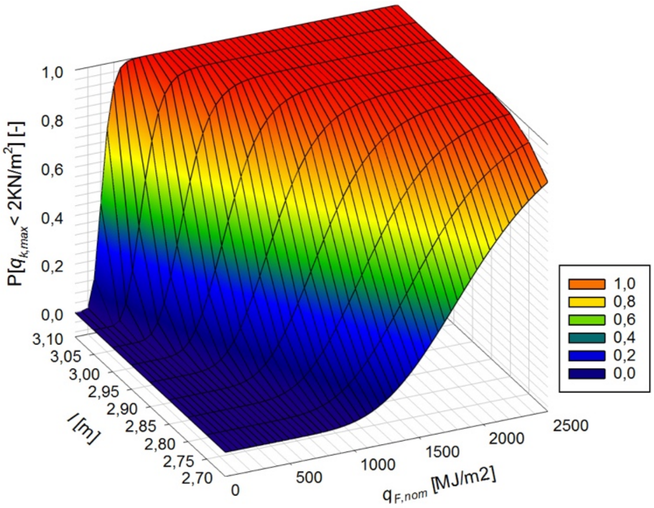

Figure 6: Fragility surface for $q_{k,req}$ = 2kN/m^2 as a function of $q_{F,nom}$ and the cantilever length l.

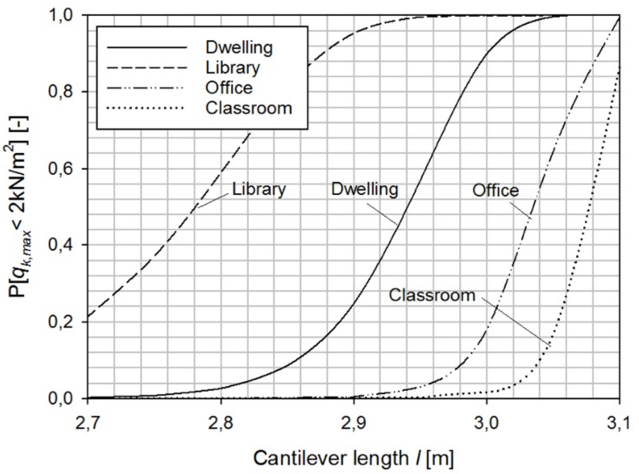

Figure 7: Fragility curves for different occupancy types as a function of the cantilever length l.

6. CONCLUSIONS

Contemporary society increasingly expects structures to be resilient in the face of extreme events. For structural fire safety engineering this implies an increased focus on post-fire usability and reparability. It is hypothesized that small investments or design changes in the construction phase could result in significant benefits for post-fire performance and use. This concept is denoted as ‘design for post-fire use’, and the general feasibility and quantification of this concept have been illustrated in this paper using an example of a concrete cantilever slab subjected to a fully developed compartment fire from below.

Considering the post-fire load bearing capacity of the slab in a multi-story apartment building, the evaluations indicate that a minor reduction in cantilever length from 3m to 2.8m could reduce the probability of safety-based use restrictions after a fire from 0.90 to less than 0.03 in this case.

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